REPORT OF
GEOTECHNICAL STUDY

PROPOSED CINCINNATI VAMC COMMUNITY LIVING CENTER
PHASE 1
3200 VINE STREET
CINCINNATI, OHIO 45220

FOR

JOHN POE ARCHITECTS

DECEMBER 2008

H. C. NUTTING

A Terracon COMPANY

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A Terracon COMPANY

HCN Division Office
611 Lunken Park Drive
Cincinnati, OH 45226
P: 513.321.5816
F: 513.321.0294
www.hcnutting.com
www.terracon.com

December 22, 2008

N1085608

Mr. John Westenkirchner
John Poe Architects
116 East Third Street
Dayton, Ohio 45402

**Re: Proposed Cincinnati VAMC Community Living Center Phase 1
3200 Vine Street
Cincinnati, Ohio 45220**

Dear Mr. Westenkirchner:

The H. C. Nutting Company / A Terracon Company is pleased to submit our report of geotechnical study for the proposed Community Living Center Phase 1 at the VA Hospital Medical Center in Cincinnati, Ohio. This geotechnical study was performed in general accordance with our proposal dated November 17, 2008 and your written authorization on November 18, 2008. The scope of this geotechnical study included characterization of existing subsurface conditions within the area of the proposed Phase 1 footprint by reviewing H. C. Nutting archives and performing a total of three Standard Penetration Test borings, laboratory examination and testing of representative samples, engineering analyses, development of geotechnical recommendations, and preparation of this report.

Based on the encountered conditions and structural load information, a deep foundation system consisting of either driven piles or augered and pressure-grout injected (auger-cast) piles is recommended. The floor slab can be designed as conventional slab-on-grade following some modification of the existing uncontrolled fill and stiffening of the slab.

Geotechnical design and construction recommendations have been provided for auger-cast pile foundations. Should vibrations and noise from driving piles be acceptable, we can provide recommendations for driven piles upon request. Recommendations for site preparation, temporary excavations, engineered fill/wall backfill placement, design and construction of subsurface walls and floor slab are provided in this report.

H. C. Nutting appreciates this opportunity of providing our professional geotechnical services for this project, and we will be glad to answer any questions concerning this report. H. C. Nutting respectfully requests continued involvement during the construction of the addition by providing testing and monitoring services as an extension of this study.

Thank you for your consideration.

Respectfully submitted,
H. C. NUTTING COMPANY

Jeffrey D. Dunlap

Jeffrey D. Dunlap, P.E.
Senior Geotechnical Engineer

Swaminathan Srinivasan

Swaminathan Srinivasan, P.E.
Senior Principal - Chief Engineer

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1.0 INTRODUCTION

H. C. Nutting / A Terracon Company was retained by John Poe Architects to perform a geotechnical study for the proposed Phase 1 Community Living Center at the VA Hospital Medical Center in Cincinnati, Ohio. The purpose of this geotechnical study was to characterize the subsurface conditions across the footprint of the proposed Phase 1 structure by performing three Standard Penetration Test (SPT) borings, and to develop geotechnical recommendations relating to the design and construction of the building. In addition, information developed from previous test borings by H. C. Nutting was reviewed and used to supplement the data as appropriate.

The following text describes our understanding of the project, our findings, and geotechnical recommendations. Following the text is an Appendix which contains figures, test boring logs, and laboratory test results. Also, included in the Appendix are descriptions of the terminology used in the test boring logs, important information regarding the basis and limitations of this study, and the storage of soil samples.

2.0 PROJECT DESCRIPTION

The proposed addition will be at the VA Hospital Medical Center located at 3200 Vine Street, in Cincinnati, Ohio (Figure 1). Information provided by the project architect (John Poe Architects) indicates that the Phase 1 Community Living Center will be located south of the existing Outpatient Clinic building (Figure 2). Based on the provided information, the proposed Living Center will consist of a three story structure with a full basement. However, only the basement and first floor will be constructed as part of the Phase 1 work. The second and third floors will be constructed as part of future phases of the development. According to the provided plans, the irregular-shaped Living Center will have approximate maximum dimensions of 125 ft. in a north-south orientation by 165 ft. in an east-west orientation. In the center of the Living Center, a courtyard with a fountain and landscaping will be installed. Additional site development will also include perimeter planting areas as well as reconfigured parking areas and access drives to the south of the proposed center. Site plans also depict an attached building just south of the Living Center, referred to as Phase 3 and 4. The Phase 1 building basement and first floor elevations are reportedly 714'-11" and 728'-0", respectively. The full basement portion of the building is located in approximately the western third of the building. The eastern portion of the building will contain a crawl space for piping and have finish elevations of 719'-4" to 722'-0".

The project structural engineer (THP Limited, Inc.) has estimated the maximum interior total column loads to be about 745 kips and maximum exterior total column loads to be about 425 kips. We have assumed maximum wall loads to be on the order of about 12 kips/lf and floor slab loads to be less than 150 psf ("point" loading conditions).

The project site is relatively level and primarily consists of pavement (asphalt concrete) areas and widely scattered trees within island areas. The test boring spot elevations indicate existing grades to range from about 725 ft. to 728 ft. Existing grades gently slope from south to north. Based on the planned basement finish floor elevation, we estimate maximum excavation on the order of about 13 ft. to establish basement subgrade elevation. Based on the surface elevations of our recent test borings, maximum structural fill is estimated on the order of 3 ft.

3.0 FIELD EXPLORATION AND LABORATORY TESTING PROGRAM

3.1 Field Exploration

Three Standard Penetration Test (SPT) borings (08-1 to 08-3 on Figure 2) were drilled within/near the Phase 1 building footprint. The test borings were field located by H. C. Nutting personnel using tape measure methods referencing from existing site features. The test borings were located within parking spaces to avoid electric lines within the parking islands and to avoid blocking the drive lanes. The spot elevations of the test borings were determined by standard level survey methods. The manhole rim located west of the proposed Phase 1 building footprint (Elev. = 726.88') was used as a temporary benchmark.

The test borings were drilled using a track-mounted drill from December 8 to December 11, 2008. The test borings were drilled to depths ranging from 89.5 ft. to 99.5 ft. below existing grade and were terminated in shale bedrock. The exploration depth at each of the borings is summarized in the table below.

Boring	Exploration Depth (ft.)
08-1	89.5
08-2	94.5
08-3	99.5

Sampling of the overburden soils was accomplished in accordance with the "Standard Method for Penetration Test and Split-Barrel Sampling of Soils" (ASTM D 1586). Split-spoon samples were typically obtained at 2.5 ft. intervals up to 20 ft. and at 5 ft. intervals thereafter. Upon reaching the shale bedrock, a sample of the bedrock was obtained by overdriving the split-barrel sampler; no rock coring was performed.

Water level observations were made during and immediately after drilling at all the test borings. Prior to demobilization, the test borings were backfilled with drill cuttings. In addition, the surfaces of the borings were patched with quick setting concrete.

3.2 Laboratory Testing Program

Upon completion of the test borings, all collected soil samples were returned to our Soil Mechanics Laboratory. Each sample was examined and visually classified in general accordance with the Unified Soil Classification System (USCS) procedures. Test boring logs were prepared by the writer based on visual examination, the drill foreman's field notes, and the results of the laboratory tests. Laboratory tests on selected soil samples included natural moisture content, loss-on-ignition and Atterberg Limit determinations, and pocket penetrometer readings (estimate of the unconfined compressive strength).

The test boring logs, the laboratory test data, details describing the test boring procedures, and the significance of the laboratory data are included in the Appendix of this report.

4.0 ENCOUNTERED SUBSURFACE CONDITIONS

The results of the three test borings (08-1 through 08-3) and four test borings performed in 2002 for the primary care addition located immediately north of the Phase 1 community living center (VA-1 through VA-4) are summarized on the Summary of Geotechnical Data sheet (Figure 4). The graphically represented logs provide generalized descriptions of the encountered soils, depths of stratum change, and the results of our laboratory tests. Detailed descriptions of the soil samples, penetration test field data, and laboratory test results are provided on the test boring logs.

The test borings typically encountered existing fill underlain by natural overburden soils consisting of glacial till interbedded with sand/silt layers, lakebed clay, and residual soils. The fill and overburden soils were underlain by gray shale bedrock. The following sections provide a brief description of the pertinent physical characteristics of each major stratum encountered in this exploration (test borings 08-1 through 08-3) in order of increasing depth below existing grade.

4.1 Pavement

All three test borings encountered asphalt concrete beginning at the ground surface. The thickness of the asphalt concrete was approximately 6 inches in each test boring. Immediately underlying the asphalt concrete, approximately 6 inches of granular base was encountered in each test boring. These thicknesses were measured by the driller and no pavement cores or samples of the granular base were obtained.

4.2 Existing Fill

All three test borings encountered existing fill soils. The fill thickness varied from about 7 ft. to 57 ft. at the test boring locations. Based on the 1912 Topographic Data (Figure 3), it appears that existing fill soils should increase in depth from the northeast portion of the proposed Phase 1 building to the southwest corner of the Phase 1 building. The approximate depth/bottom elevation of the fill encountered at the test borings is summarized in the table below.

Boring	Surface Elevation (ft.)	Fill Thickness (±ft.)			Fill Bottom Elevation (±ft.)
		Granular	Cohesive	Total	
08-1	728.1	50.0	7.0	57.0	670.1
08-2	727.0	8.0	9.0	17.0	709.0
08-3	725.4	---	7.0	7.0	717.4

The cohesive fill has typically been described as brown/brown and gray lean clay, with sand, gravel, rock, brick, asphalt and glass fragments in various proportions. The fill was typically moist and of stiff to very stiff consistency (noted some soft and hard zones). Moisture content of tested fill samples ranged from 13% to 27% being typically in the teens and mid 20s. Pocket penetrometer readings ranged from 0.5 tsf to in excess of 4.5 tsf, being typically less than 2.0 tsf.

The granular fill has been described as brown to black cinders with sand and noted glass, metal, slag and brick fragments in various proportions. The granular fill was moist and of very loose to loose compactness. The SPT "N-Value" (blows per foot) ranged from 3 to 7 blows/ft. and was typically less than 4 blows/ft. A 5 ft. thick zone of granular fill in test boring 08-1 beginning at a depth of 48 ft. below grade was described as wet.

Based on the observed variations in fill composition, moisture content, and consistency/compactness, we have interpreted this fill to be uncontrolled. We have not reviewed field inspection reports for the placement of this fill to confirm its placement in a controlled manner.

4.3 Natural Overburden Soils

Underlying the existing fill, the test borings encountered natural overburden soils consisting of glacial till, lakebed clay and residual clay soils. Immediately underlying the fill, the borings typically encountered brown glacial till described as a lean clay with sand and gravel in various proportions. The brown glacial till is a result of the leaching and weathering of the underlying gray glacial till. The brown glacial till was typically moist and of stiff to very stiff consistency with occasional medium stiff zones. Moisture contents of tested samples ranged from 14% to 23%. Pocket penetrometer readings ranged from 0.7 tsf to 4.0 tsf being typically more than 2 tsf.

Underlying the brown glacial till stratum, the borings encountered gray glacial till and gray lakebed clay soils. The gray glacial till and lakebed soils have typically been described as a lean clay (some fat clay). This stratum was typically moist to very moist (some wet zones) and of soft to very stiff. Moisture content of tested samples ranged from 9% to 31% being typically in the teens to mid-20's. Pocket penetrometer readings ranged from 0. 5 tsf to 3.2 tsf being typically between 0.5 tsf and 1.5 tsf. Occasional wet to very moist sand and or silt seams to partings were encountered in the glacial till soils. The presence of wet sand/silt seams and layers is common in glacial till.

Atterberg Limits of a sample gray glacial till from 08-3 indicated the Liquid and Plastic Limits to be 28% and 16%, respectively. Per the USCS classification, this sample would classify as a "Lean Clay (CL)". Atterberg Limits of a lakebed clay sample from 08-3 indicated the Liquid and Plastic Limits to be 45% and 22%, respectively. Per the USCS classification, this sample would classify as a "Lean Clay (CL)".

A deeper layer of gray glacial till was encountered between the residual soil and the lakebed soil in test boring 08-2 beginning at a depth of 88 ft. below existing grade. The thickness of the deeper glacial till layer was 5 ft. in test boring 08-2.

The borings encountered residual clay soils immediately over the gray shale bedrock. These soils are formed by the complete weathering of the underlying parent bedrock. The thickness of this stratum ranged from 3 ft. to 5 ft. This stratum was moist to very moist (some wet zones) and of medium stiff to very stiff consistency. Moisture content of two tested samples ranged from 18% to 24%. Pocket penetrometer readings ranged from 2.0 tsf to 3.5 tsf.

4.4 Shale Bedrock

Underlying the fill and natural overburden soils, the test borings were terminated in gray shale bedrock of Ordovician Age. The depth/elevation at which the gray shale bedrock was encountered at the recent and 2002 borings is summarized in the table below.

Boring	Surface Elevation (ft.)	Gray Shale Bedrock	
		Depth (±ft)	Elevation (±ft.)
08-1	728.1	83.0	645.1
08-2	727.0	98.0	629.0
08-3	725.4	93.0	632.4
VA-1	725.5	60.5	665.0
VA-2	726.3	75.0	651.3
VA-3	726.8	90.0	636.8
VA-4	726.7	105.0	621.7

The approximate surface of gray shale bedrock contours have been interpolated using available test boring data and are shown on Figure 5 in the Appendix. It should be noted that these contours represent the approximate surface and field variations should be anticipated due to natural depositional and erosion processes. The bedrock is observed to dip from northeast to southwest at approximately 4H:1V and then rises at an estimated 2.5H:1V toward the southwest corner of the Phase 1 building area. Actual top of bedrock data is unavailable at the southwest corner of the Phase 1 building area and the dashed bedrock contours should be considered very approximate at best. Additional test borings would be required to more accurately estimate the top of bedrock surface.

Based on the elevation at which the bedrock was encountered, published geologic literature cites that the bedrock is of Ordovician Age and primarily belongs to the Kope Formation of the McMicken Member.

4.5 Groundwater

Groundwater conditions were observed at all the borings during drilling and immediately upon completion of drilling. The elevation at which groundwater seepage was encountered in any borehole during the drilling program is indicated on the Summary of Geotechnical Data sheet (Figure 4). During drilling, water was encountered at depths between 43 ft. and 48 ft. below grade. Upon completion of drilling, no free ground water was observed in the boreholes.

These short-term water level observations are inadequate to establish the long-term, static groundwater table. It should be noted that "trapped/perched" water conditions may be present within the existing fill and the glacial till soils. From experience, seepage is commonly observed at the fill/overburden soil interface. The glacial till soils commonly contain saturated silt, sand, or gravel pockets, seams, or layers sandwiched between less permeable cohesive soils. Also, long-term static water levels are known to occur near the brown/gray glacial till interface. The gray glacial till and the sand/slit seams were encountered between approximate elevations 665 and 697. Long-term water levels will also vary with rainfall, and other seasonal variations.

Based on our short-term observations during drilling, it is our opinion, any seepage that may be encountered within the basement and crawl space excavations can most likely be handled by conventional sump-pumping or gravity drains.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 General Assessment

The test borings reveal a subsurface profile consisting typically of about 7 ft. to 57 ft. of uncontrolled fill, underlain by natural overburden soils including glacial till, lakebed clay, and residual soils and shale bedrock. The existing uncontrolled fill is unsuitable for direct foundation and floor slab support. The natural overburden soils are observed to generally consist of an upper moist, very stiff crust/stratum underlain by moist to very moist and soft to stiff soils. The competency/consistency of the natural overburden soils is generally observed to diminish with depth. Based on the estimated loads and the encountered subsurface conditions, we recommend a deep foundation system bearing in the gray shale bedrock for structure support to limit total and differential foundation settlements within tolerable limits.

Based on the anticipated pile lengths and discussions with a local foundation contractor, installation of drilled piers is anticipated to be cost prohibitive. Driven steel H-piles could be considered; however, the impact of vibrations during pile driving on existing facilities should be evaluated. Auger-cast piles can be considered for the foundation system. The vibrations during installation of auger-cast piles are typically minimal/tolerable. However, with auger-cast piles it is extremely important that the feasibility/constructability of reinforcing the upper portion of each pile in accordance with the requirements of Section 1810.3.4 and 1810.3.5 of the 2007 Ohio Building Code (OBC) be evaluated prior to final foundation design. Based on our past experience at the project site, it may be feasible to insert the reinforcing cage to a depth of about 30 ft. to 35 ft. below pile top. Once the pile reinforcement requirements are evaluated, we strongly recommend that an experienced foundation contractor be consulted (prior to bidding) to evaluate the constructability. The recommendations in this report are specifically for an auger-cast pile foundation system. Recommendations for driven H-piles bearing on bedrock can be provided upon request as an addendum to this report.

It is anticipated that the proposed basement and crawl space excavations can be performed as open cut excavations. However, the feasibility of performing open cut excavations will be impacted by location/proximity of underground utilities that will remain and established excavation limits. The basement floor slab can be designed as conventional slab-on-grade.

Specific recommendations for site preparation, temporary excavation, engineered fill/wall backfill placement, and design and construction of auger-cast piles, subsurface walls and floor slab are detailed in the following paragraphs.

5.2 Site Preparation and Engineered Fill Placement

Site preparation is anticipated to consist of relocation of existing utilities, stripping of topsoil/mulch and grubbing of shrubs/trees in planter areas, and removing existing asphaltic concrete pavements within the building footprint. The stripped topsoil/mulch is unsuitable for reuse in engineered fill and can be stockpiled for later use for landscaping purposes. The existing shrubs and trees within the addition footprint should be cleared, grubbed (including roots), and backfilled (as necessary) with new engineered fill placed in accordance with recommendations in this section. The construction debris and asphalt pavements within the building addition footprint should be removed and wasted off-site.

As indicated earlier, we anticipate the basement and crawl space excavations can be performed as open cut excavations. Following excavation to subgrade elevation, the basement area should be proof rolled under close monitoring by geotechnical personnel to identify/isolate soft, yielding areas. The proof rolling should be accomplished with several passes of heavy construction equipment such as a fully loaded tandem axle truck or sheeps foot roller. Soft, yielding areas should be evaluated by the geotechnical technician, undercut as necessary and backfilled with new engineered fill placed in accordance with our recommendations for engineered fill placement. In areas of existing uncontrolled fill, we recommend a minimum 2 ft undercut and the subgrade elevation can be reestablished with engineered fill.

It is recommended that the engineered fill soils required to reach subgrade elevation for the floor slab be placed in maximum 8" loose, horizontal lifts and compacted to at least 98% Standard Proctor maximum dry density, (ASTM D 698). The fill soils for general site grading purpose (non-structural areas) could be compacted to 95% of Standard Proctor maximum dry density, provided future structural developments are not planned.

All engineered fill material should consist of a relatively clean soil, free of organics, debris, and other deleterious substances. Where a cohesive fill is used, its plasticity index should be less than 25. The maximum particle size of the fill material should be less than 4" in any dimension. The suitability and laboratory compaction characteristics of engineered fill soils should be determined prior to use.

Based on a review of the laboratory jar samples, it is anticipated that at least 50% of the excavated fill soils would be generally suitable for reuse as engineered fill and will need to be moisture conditioned prior to reuse. The extent of moisture conditioning required will depend on several factors including stockpile duration and climatic conditions during stockpiling. The preferred material for reuse as structural fill is the cohesive soils having lower moisture contents. The use of the cinder fill soils as new structural fill should not be considered based on the 15% loss-on-ignition value of a tested sample. Existing uncontrolled fill should be further evaluated in the field and laboratory for its suitability as engineered fill prior to reuse.

5.3 Temporary Excavations and Retention

Excavation on the order of about 13 ft. is anticipated to establish the basement floor slab subgrade and excavations up to 8 ft. are anticipated in the crawl space areas that will house piping. The feasibility of performing open-cut excavations depends on the proximity of existing structures and utilities to remain and the established excavation limits. Based on the information on the site plan provided to us and our understanding of the existing Outpatient Clinic structure, we do not anticipate the need for temporary retention along the northern perimeter of the addition. It appears that open cut excavation can likely be performed along the perimeters of the basement and crawl space footprint. However, the feasibility of performing open cut excavations will be impacted by the proximity of existing utilities to remain and the established excavation limits.

Open cut excavations deeper than 4 ft. should be performed in accordance with OSHA Excavation Regulations. Per the OSHA Soil Classification, the existing uncontrolled fill soils range from Type "B" to Type "C" and we recommend that excavation slopes within these soils be no steeper than 1.5H:1V. The temporary excavation slopes should be examined periodically to evaluate any potential destabilizing effects due to surface water erosion or subsurface seepage conditions which are typically more prevalent during wetter seasons of the year. Some sloughing of the excavation side slopes may occur, which would require redressing or removal. All excavations greater than 20 ft. should be designed by a registered geotechnical engineer.

If any temporary retention is required, the actual temporary retention system should be designed by a specialty contractor, with the design being reviewed by the geotechnical and structural engineer. There should be a performance criteria specifying how much lateral movement can be tolerated so that the retention system is designed to meet this criteria. The typical allowable movement which we have seen in the past is ½" to 1". The amount of allowable movement is dependent on the proximity of existing structures and utilities and should be stipulated by the structural engineer.

5.4 Auger-Cast Pile Design and Construction

Auger-cast piles end bearing in the gray shale and limestone bedrock are recommended for foundation support. The recommended allowable design axial capacity and estimated range of tip elevations for 16", 18", and 24" diameter auger-cast piles are summarized below. Please note that the length of the auger-cast piles will change across the foot print of the building. Figure 5 in the appendix can be used to aid in estimating the proposed pile lengths; however, variations between design lengths and actual auger-cast pile lengths should be expected. Our past experience indicates that the auger-cast piles are capable of being installed through the encountered overburden soils allowing the pile tips to bear on the shale and limestone bedrock.

Pile Diameter (in.)	Allowable Axial Pile Capacity[1] (kips)	Tip Elev./Length[2] Range (±ft.)
16	200	620 to 650/ 90 to 60
18	250	620 to 650/ 90 to 60
24	450	620 to 650/ 90 to 60

1. Requires grout with minimum 28-day compressive strength of 4,000 psi.
2. Measured from assumed bottom of pile cap elev. of 710±

In addition to the design capacities summarized above, the allowable design stress should not exceed 25% of the 28-day grout compressive strength, based on our past experience with 16" diameter auger-cast piles at the site and previous pile load testing on 16" diameter auger-cast piles at the site. If auger-cast piles having a diameter other than 16" are desired, static pile load testing will be required for these piles. If additional pile load testing is performed at the site, higher pile capacities based upon allowable design stress not exceeding 33% of the 28-day grout compressive strength per OBC section 1810.3 of the 2007 OBC could be realized. A grout mix having a 28-day compressive strength of at least 4,000 psi is recommended.

The piles have been designed to develop their capacities by bearing on the shale bedrock and should be installed to a condition of practical auger refusal on unweathered gray shale bedrock. The pile contractor should use a drill rig having a minimum 100,000 foot-pound torque and crowding capabilities. Refusal criteria should consist of auger penetration of 6 inches or less per minutes, or as otherwise determined by the owner's geotechnical consultant based on review of the load test data.

Piles should be spaced no less than 3.0 pile diameters center-to-center, and a group efficiency of 1.0 (for axial compressive loads) can be used for design of pile groups. Settlement for pile groups should be small, approximating the theoretical elastic compression of the pile member plus 1/4".

The allowable lateral load for the 16", 18" and 24" diameter piles under "fixed head" conditions is estimated to be 6 kips, 8 kips, and 12 kips per pile, respectively, with lateral deflection of 0.25"; appropriate reinforcing should be included. Once the pile configurations, pile head fixity, and lateral loads are known, a detailed lateral analysis can be performed as an addendum to this report and an addition to our current scope. Based on pile spacing, group effects for lateral loads can be evaluated. Assuming a center-to-center pile spacing of 3 pile diameters, a group reduction factor of 0.7 (i.e. individual capacity x 0.7) is recommended for lateral loads.

For pile groups, the passive resistance of existing uncontrolled fill soil against the pile cap cannot be relied upon in design. For pile caps in the stiff to very stiff, natural brown glacial till, a uniform allowable passive resistance of 750 psf across the face of the cap can be used in design provided the pile cap excavations are neat and concrete is placed directly against the excavation face; no forming should be allowed. The evaluation of lateral resistance should not include the friction between the bottom of pile cap and underlying subgrade soil, since these soils may be disturbed during construction and cannot be relied upon to maintain contact with the bottom of pile cap.

Auger-cast piles should be installed by an experienced specialty contractor. It is recommended that a full-length reinforcing bar, No. 9 bar or larger, centered within the pile be required. Centralized bar placement should be feasible without the use of centralizers if a bottom discharge bit is used. Supplemental reinforcing within the top portion of the pile should be included, as may be necessary, considering structural requirements to address bending moments and shear forces. The steel reinforcement of the auger-cast piles should be in accordance with Section 1810.3.4 and 1810.3.5 of the 2007 OBC. Once the reinforcement requirements for the Seismic Design Category (SDC) are evaluated, it is extremely important that its constructability be evaluated by an experienced foundation contractor.

The specifications should require that the total grout volume in each pile be at least 120 percent of the theoretical "neat-line" pile volume. In accordance with Section 1810.3.3 of the 2007 OBC, the piles shall not be installed within 6 pile diameters center-to-center of a pile grouted less than 12 hours old. Full-time inspection by geotechnical personnel is necessary during pile installation to monitor plumbness, grouting procedures, sample the grout, monitor the auger withdrawal rate during grouting, placement of reinforcing steel/cage, and other elements critical to the finished pile structure.

When pile design capacities exceed 40 tons (80 kips) per pile (which have been recommended), then at least one pile load test will be required, per Section 1808.2.8 of the 2007 OBC. If the 16" diameter auger-cast piles with an allowable axial design capacity of 200 kips are selected, no additional pile load testing will be required, since the previous load test confirms this allowable axial design capacity. If any other auger-cast pile diameters or a larger allowable capacity is desired for the 16" diameter auger-cast piles, at least one new static pile load test will be required. Our office should be consulted in planning, performance, and analysis of the load test. For general purposes, it is recommended that the pile be tested in accordance with ASTM D 1143, per the "Quick Method." It is recommended that the test pile be loaded either to 250%

of design load or failure whichever comes first. We recommend that the specifications require the successful completion of the load test prior to the installation of production piles.

The test pile should not be used for a production pile. However, reaction piles can be considered for use as production piles. Also, grout cubes should be tested prior to pile load testing, in order to confirm adequate compressive strength. The test pile should be constructed using the same equipment, grout quantity, and other features proposed for the production piles. It is recommended that H.C. Nutting be allowed to review the contractor's proposed pile load test program (layout, loading schedule, etc.) prior to the test.

5.5 Basement and Crawl Space Floor Slab

The basement and crawl space floor slab subgrade is anticipated to consist of a combination of existing natural, brown glacial till and uncontrolled fill soils. The existing uncontrolled fill is not considered a suitable subgrade material. It is recommended that the subgrade consist either of natural glacial till soils or at least 2 ft. of new engineered fill. The 2 ft. of new engineered fill is to provide more uniform floor slab support. Due to the presence of existing uncontrolled fill, it is recommended that closer than normal construction joints be considered. Additionally, the stiffness of the floor slab could be increased either by increasing the floor slab thickness or using additional reinforcement.

In uncontrolled fill areas, the undercut footprint should be proof rolled prior to placement of the new engineered fill to establish floor slab subgrade. Proof rolling should be performed, under close monitoring by geotechnical personnel, with several passes of heavy construction equipment to identify soft/yielding areas. Soft/yielding areas should either be stabilized in place or be undercut and replaced with engineered fill in accordance with recommendations in this report. The floor slab subgrade should be proof rolled immediately prior to placement of the granular base and floor slab concrete.

Support of floor slabs on or above existing fill soils is discussed in this report. However, even with the recommended construction testing services, there is an inherent risk for the owner that compressible fill or unsuitable material within or buried by the fill will not be discovered. This risk of unforeseen conditions cannot be eliminated without completely removing the existing fill, but can be reduced by performing additional testing and evaluation.

For "point" loading conditions, we recommend that the building floor slab design be based on a modulus of subgrade reaction (k_s) value of 100 pci. This is provided the subgrade soils are prepared in accordance with our recommendations.

It is recommended that a 6" thick compacted granular base consisting either of ODOT Item 304 crushed limestone, sand and gravel or approved equivalent be placed between the approved subgrade and the floor slab bottom. This granular base will serve as a leveling course and help achieve a more uniform slab thickness, provide more uniform load transfer, and aid in curing of the concrete at both the top and bottom of the floor slab. It is also recommended that a vapor barrier be placed between the prepared subgrade and the floor slab in areas where a floor covering such as tile, carpet etc. is planned.

It is recommended that all interior utility trenches (typically backfilled with granular material to meet compaction requirements) be sloped to develop positive gravity flow for any water which would otherwise accumulate within the trenches. The collected water should be directed to a point of discharge or collection system outside the building limits. Additionally, we recommend that the trenches be bulkheaded with cohesive material or lean concrete at upslope building entry/exit points to minimize outside water from infiltrating the granular backfill.

5.6 Basement and Subsurface Walls

Permanent subsurface walls with a height of about 13 ft. are anticipated in the basement area of the addition. The subsurface walls for the crawl space areas will likely have maximum heights of approximately 9 ft. The magnitude and distribution of the lateral earth pressure depends on the wall type, size, degree of restraint against rotation at its top, surcharge load and distribution, backfill type, and compaction. It is presumed that the basement and crawl space walls will be somewhat rigid and unyielding (since their top will be tied into the floor slab). We recommend that the basement and crawl space walls be designed using "at-rest" earth pressure conditions. We recommend the following lateral earth pressures be used for wall design.

Horizontal "At-Rest" Pressure: 30 H lbs/sq. ft. (Rectangular Distribution) plus one-half of any surface surcharge loading (S/2) 'H' is the retained soil height in ft. and 'S' is surface surcharge load in psf

The lateral earth pressure value is based upon the assumption that the basement/subsurface walls have a relatively level backfill, are backfilled with a zone of free-draining granular material at least 3 ft. wide (measured horizontally from the wall face) and that positive drainage is provided. It is important that proper drainage be provided by foundation drains so that hydrostatic pressures do not develop as the resulting lateral pressures would otherwise be substantially higher than those recommended above.

Backfill against the basement and crawl space walls should consist of a free-draining granular material, having no more than 7% passing the No. 200 sieve. The granular backfill should be placed in 4" to 6" thick loose horizontal lifts with each lift being compacted to 98% of the Standard Proctor maximum dry density (ASTM D 698). In order to avoid overstressing of the wall, hand compaction equipment should be used within 5 ft. of the wall's face, and use of heavy compaction equipment should be avoided near the wall. To avoid surface water runoff from directly infiltrating the granular backfill, a layer of cohesive soil (12" to 18" thick) or paving should cap the surface of the granular backfill, and exterior grades should be sloped away from the walls.

5.7 Flexible Asphalt Pavement

The pavement subgrade is anticipated to consist of predominantly uncontrolled fill soils. The existing uncontrolled fill is not considered a suitable subgrade material. It is recommended that the subgrade consist either of at least 2 ft. of new engineered fill. The 2 ft. of new engineered fill will provide more uniform pavement support. Prior to placement of the new structural fill (where required), the exposed subgrade should be proof rolled and any soft or disturbed areas should be either stabilized or undercut until stable soils are encountered. Again, soft, wet soils or organic soils should be undercut prior to placing new structural fill.

Support of pavements on or above existing fill soils is discussed in this report. However, even with the recommended construction testing services, there is an inherent risk for the owner that compressible fill or unsuitable material within or buried by the fill will not be discovered. This risk of unforeseen conditions cannot be eliminated without completely removing the existing fill, but can be reduced by performing additional testing and evaluation.

Design of asphaltic concrete paving can be based on a CBR value of 3 and a resilient modulus of 3600 psi. For rigid concrete pavement, a modulus of subgrade reaction of 100 pci can be used for the design. We recommend that the entire pavement areas also be proof rolled with heavy construction equipment immediately prior to paving operations to determine the presence of any soft or yielding surface soils.

In all cases, the pavement should be constructed in accordance with ODOT Specifications. Sufficient drainage should be provided both at the pavement surface and at soil subgrade level. If any surrounding ground surface slopes down towards the pavement, an interceptor ditch or edge drain is recommended to intercept potential seepage that could enter the pavement base and subgrade.

The long-term performance of flexible asphalt pavements will depend significantly on proper subgrade preparation and effective drainage. Adequate drainage facilities are necessary to promote positive drainage of surface and subsurface water around and below pavements. The goal of such drainage should be to effectively collect surface water from the parking lots, the drives, and other paved areas while discharging the water at a suitable outlet or into a storm sewer. Specifically, we recommend that drainage be diverted away from parking areas and structures. Curbs, drain tiles, catch basins, intercepting swales, and storm sewers should be constructed to provide positive drainage from the site. In addition, pavement edge drains or intercepting swales are recommended for areas where any surface grades will be directed toward the paved areas. The inverts of the intercepting swales or pavement edge drains should be extended lower than any pavement subgrade. The pavement should be constructed in accordance with current ODOT Specifications.

5.8 Seismic Site Classification

Section 1615 of the 2007 Ohio Building Code (OBC) recommends that every structure be designed and constructed to resist the effects of earthquake motions. The test borings in this study encountered varying depths of existing uncontrolled underlain by natural overburden soils and gray shale bedrock belonging to the Kope formation. The depth to bedrock below existing grades ranged from about 80 ft. toward the west to about 98 ft. toward the east. The shear wave velocity for the gray shale bedrock has been estimated from limited laboratory tests by H.C. Nutting in the Kope formation. The shear wave velocity of the overburden soils has been estimated from the observed consistency/undrained shear strength of the overburden soils (estimated from pocket penetrometer readings) and published correlation between the shear wave velocity and the undrained shear strength of soils. The weighted average shear wave velocity in the top 100 ft. of the soil/bedrock profile at this site is estimated to be between 600 ft/sec. and 900 ft/sec. Hence, a Site Class "D" (in accordance with Table 1615.1.1 of the 2002 OBC) is recommended to determine the Seismic Design Category (SDC) for structural design and detailing. Due to the cohesive nature of the overburden soils, liquefaction is not a significant concern.

6.0 CONSTRUCTION TESTING AND MONITORING

It is recommended that during construction, close monitoring be performed by a geotechnical technician working under the direction of the Project Geotechnical Engineer. This monitoring should be performed during site preparation, undercutting existing uncontrolled fill, placement and compaction of engineered fill/wall backfill, installation of auger-cast pile foundations, floor slab subgrade preparation and construction materials testing.

In our opinion, the construction testing and monitoring services should be provided by the same firm developing the geotechnical recommendations. We therefore respectfully request that H. C. Nutting be retained for this work.

H. C. NUTTING

A Terracon COMPANY

LIMITATIONS OF LIABILITY

OUR WARRANTY

We warrant that the services performed by H. C. Nutting Company are conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions NO OTHER WARRANTIES, EXPRESSED OR IMPLIED, ARE MADE. While the services of H. C. Nutting Company are a valuable and integral part of the design and construction teams, we do not warrant, guarantee, or insure the quality or completeness of services provided by other members of those teams, the quality, completeness, or satisfactory performance of construction plans and specifications which we have not prepared, nor the ultimate performance of building site materials.

SUBSURFACE EXPLORATION

Subsurface exploration is normally accomplished by test borings; test pits are sometimes employed. The method of determining the boring location and the surface elevation at the boring is noted in the report. The information is represented on a drawing or on the boring log. The location and elevation of the boring should be considered accurate only to the degree inherent with the method used.

The boring log includes sampling information, description of the materials recovered, approximate depth of boundaries between soil and rock strata and groundwater data. The log represents conditions specifically at the location and time the boring was made. The boundaries between different soil strata are indicated at specific depths; however, these depths are in fact approximate and dependent upon the frequency of sampling. The transition between soil strata is often gradual. Water level readings are made at the times and under the conditions stated on the boring logs. Water levels change with time and season. The borehole does not always remain open sufficiently long for the measured water level to coincide with the groundwater table.

LABORATORY AND FIELD TESTS

Tests are performed in accordance with specific ASTM Standards unless otherwise indicated. All determinations included in a given ASTM Standard are not always required and performed. Each test report indicates the measurements and determinations actually made.

ANALYSIS AND RECOMMENDATIONS

The geotechnical report is prepared primarily to aid in the design of site work and structural foundations. Although the information in the report is expected to be sufficient for these purposes, it is not intended to determine the cost of construction or to stand alone as a construction specification.

Report recommendations are based primarily on data from test borings made at the test locations shown on a boring location drawing included. Soil variations may exist between borings and these variations may not become evident until construction. If significant variations are then noted, the geotechnical engineer should be contacted so that field conditions can be examined and recommendations revised if necessary.

The geotechnical report states our understanding as to the location, dimensions and structural features proposed for the site. Any significant changes in the nature, design, or location of the site improvements MUST be communicated to the geotechnical engineer so that the geotechnical analysis, conclusions, and recommendations can be appropriately adjusted.

The geotechnical engineer should be given the opportunity to review all drawings that have been prepared based on his recommendations.

CONSTRUCTION MONITORING

Construction monitoring is a vital element of complete geotechnical services. The field engineer/inspector is the owner's "representative" observing the work of the contractor, performing tests as required in the specifications, and reporting data developed from such tests and observations. THE FIELD ENGINEER OR INSPECTOR DOES NOT DIRECT THE CONTRACTOR'S CONSTRUCTION MEANS, METHODS, OPERATIONS OR PERSONNEL. He does not interfere with the relationship between the owner and the contractor and, except as an observer, does not become a substitute owner on site. He is responsible for his own safety but has no responsibility for the safety of other personnel at the site. He is an important member of a team whose responsibility is to watch and test the work being done and report to the owner whether that work is being carried out in general conformance with the plans and specifications.

APPENDIX

BORING TERMINOLOGY
SOIL CLASSIFICATION
FIGURE 1: SITE VICINITY MAP
FIGURE 2: TEST BORING LOCATION PLAN
FIGURE 3: 1912 TOPOGRAPHIC SURVEY
FIGURE 4: SUMMARY OF GEOTECHNICAL DATA
FIGURE 5: APPROXIMATE TOP OF GRAY SHALE BEDROCK CONTOURS
TEST BORING LOGS
LABORATORY TEST DATA
SAMPLE DISPOSITION

A description of terminology and symbols used in the logs of test borings, and a copy of ASTM D 2487, "Classification of Soils for Engineering Purposes", are included in the following two pages.

Readers of this report who wish an in-depth discussion on the basis for geotechnics, including procedures used in subsurface exploration, laboratory testing, and geotechnical analyses are referred to The H. C. Nutting Geotechnical and Test Engineering Manual. Those readers not having a copy of this manual may obtain one at nominal cost by contacting The H. C. Nutting Company at (513) 321-5816.

GENERAL NOTES

DRILLING & SAMPLING SYMBOLS:

SS:	Split Spoon - 1-^{3}/8" I.D., 2" O.D., unless otherwise noted	HS:	Hollow Stem Auger
ST:	Thin-Walled Tube - 2" O.D., unless otherwise noted	PA:	Power Auger
RS:	Ring Sampler - 2.42" I.D., 3" O.D., unless otherwise noted	HA:	Hand Auger
DB:	Diamond Bit Coring - 4", N, B	RB:	Rock Bit
BS:	Bulk Sample or Auger Sample	WB:	Wash Boring or Mud Rotary

The number of blows required to advance a standard 2-inch O.D. split-spoon sampler (SS) the last 12 inches of the total 18-inch penetration with a 140-pound hammer falling 30 inches is considered the "Standard Penetration" or "N-value".

WATER LEVEL MEASUREMENT SYMBOLS:

WL:	Water Level	WS:	While Sampling	N/E:	Not Encountered
WCI:	Wet Cave in	WD:	While Drilling		
DCI:	Dry Cave in	BCR:	Before Casing Removal		
AB:	After Boring	ACR:	After Casing Removal		

Water levels indicated on the boring logs are the levels measured in the borings at the times indicated. Groundwater levels at other times and other locations across the site could vary. In pervious soils, the indicated levels may reflect the location of groundwater. In low permeability soils, the accurate determination of groundwater levels may not be possible with only short-term observations.

DESCRIPTIVE SOIL CLASSIFICATION: Soil classification is based on the Unified Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

CONSISTENCY OF FINE-GRAINED SOILS

Unconfined Compressive Strength, Qu, psf	Standard Penetration or N-value (SS) Blows/Ft.	Consistency
< 500	0 - 1	Very Soft
500 – 1,000	2 - 4	Soft
1,000 – 2,000	4 - 8	Medium Stiff
2,000 – 4,000	8 - 15	Stiff
4,000 – 8,000	15 - 30	Very Stiff
8,000+	> 30	Hard

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Standard Penetration or N-value (SS) Blows/Ft.	Relative Density
0 – 3	Very Loose
4 – 9	Loose
10 – 29	Medium Dense
30 – 49	Dense
> 50	Very Dense

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 15
With	15 – 29
Modifier	> 30

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300mm)
Cobbles	12 in. to 3 in. (300mm to 75 mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm)
Silt or Clay	Passing #200 Sieve (0.075mm)

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 5
With	5 – 12
Modifiers	> 12

PLASTICITY DESCRIPTION

Term	Plasticity Index
Non-plastic	0
Low	1-10
Medium	11-30
High	> 30

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]					Soil Classification	
					Group Symbol	Group Name[B]
Coarse Grained Soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$[E]		GW	Well-graded gravel[F]
			$Cu < 4$ and/or $1 > Cc > 3$[E]		GP	Poorly graded gravel[F]
		Gravels with Fines More than 12% fines[C]	Fines classify as ML or MH		GM	Silty gravel[F,G,H]
			Fines classify as CL or CH		GC	Clayey gravel[F,G,H]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$[E]		SW	Well-graded sand[I]
			$Cu < 6$ and/or $1 > Cc > 3$[E]		SP	Poorly graded sand[I]
		Sands with Fines More than 12% fines[D]	Fines classify as ML or MH		SM	Silty sand[G,H,I]
			Fines Classify as CL or CH		SC	Clayey sand[G,H,I]
Fine-Grained Soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	inorganic	PI > 7 and plots on or above "A" line[J]		CL	Lean clay[K,L,M]
			PI < 4 or plots below "A" line[J]		ML	Silt[K,L,M]
		organic	Liquid limit - oven dried	< 0.75	OL	Organic clay[K,L,M,N]
			Liquid limit - not dried			Organic silt[K,L,M,O]
	Silts and Clays Liquid limit 50 or more	inorganic	PI plots on or above "A" line		CH	Fat clay[K,L,M]
			PI plots below "A" line		MH	Elastic Silt[K,L,M]
		organic	Liquid limit - oven dried	< 0.75	OH	Organic clay[K,L,M,P]
			Liquid limit - not dried			Organic silt[K,L,M,Q]
Highly organic soils	Primarily organic matter, dark in color, and organic odor				PT	Peat

[A] Based on the material passing the 3-in. (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10} \quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ≥ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.

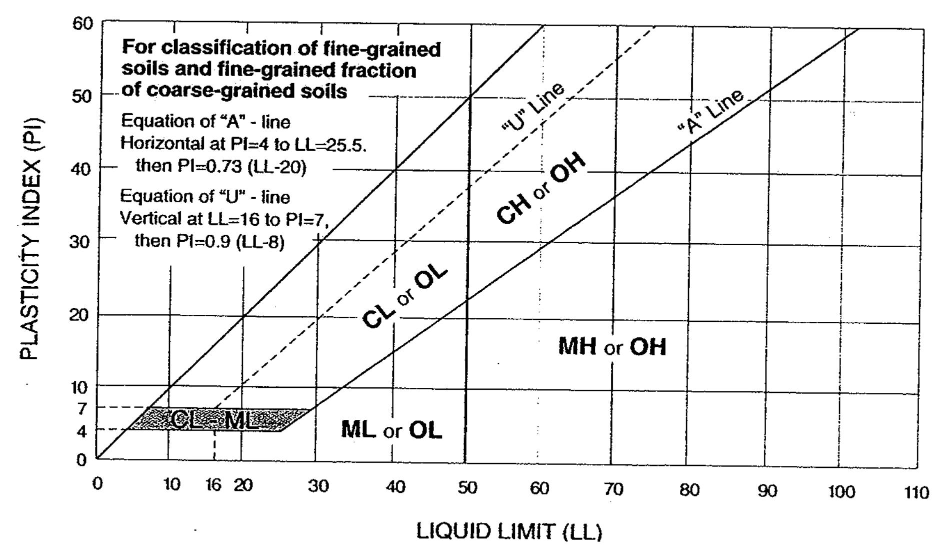
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI ≥ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

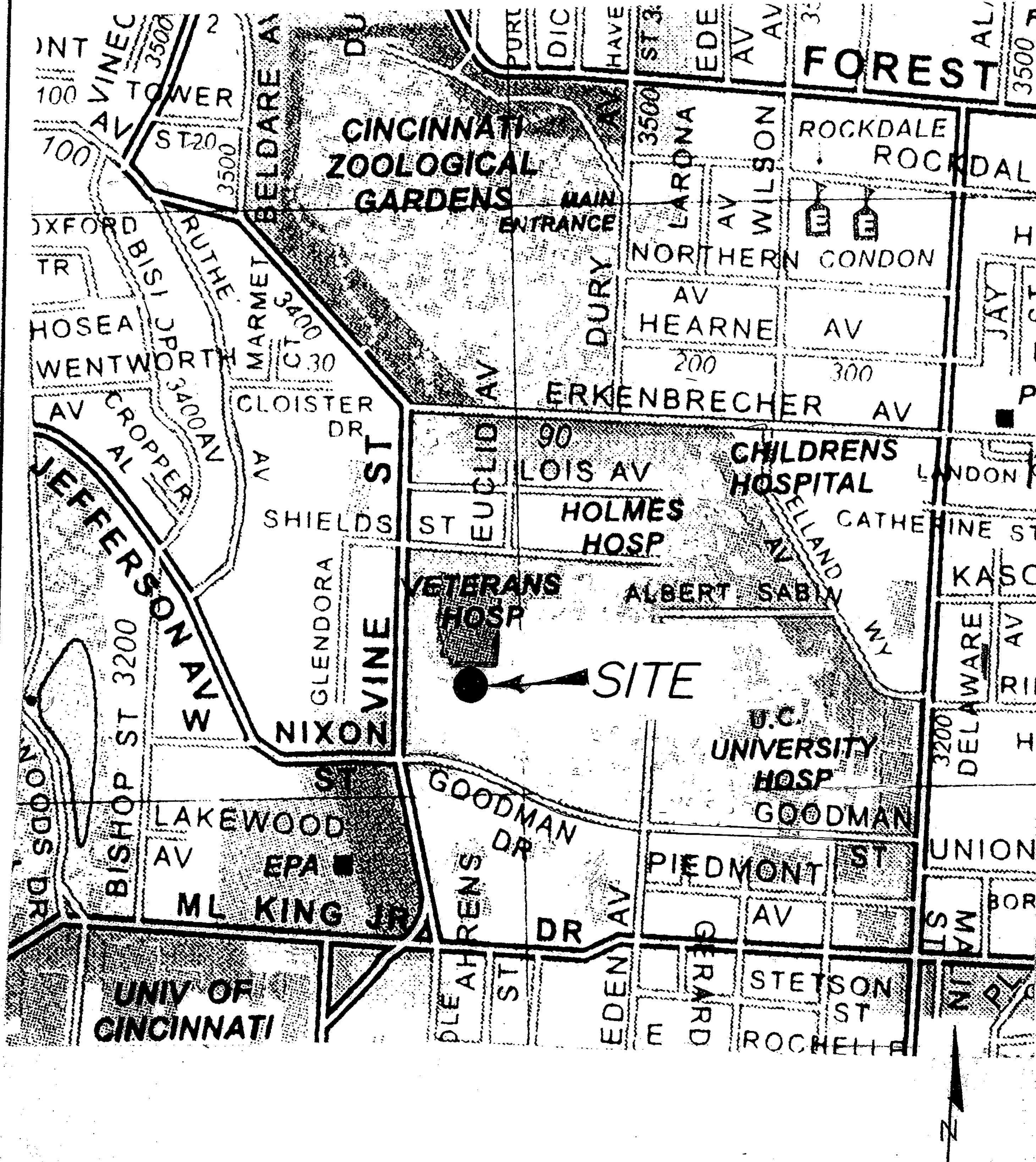
[Q] PI plots below "A" line.



H. C. NUTTING

A Terracon COMPANY

Form 111—6/98



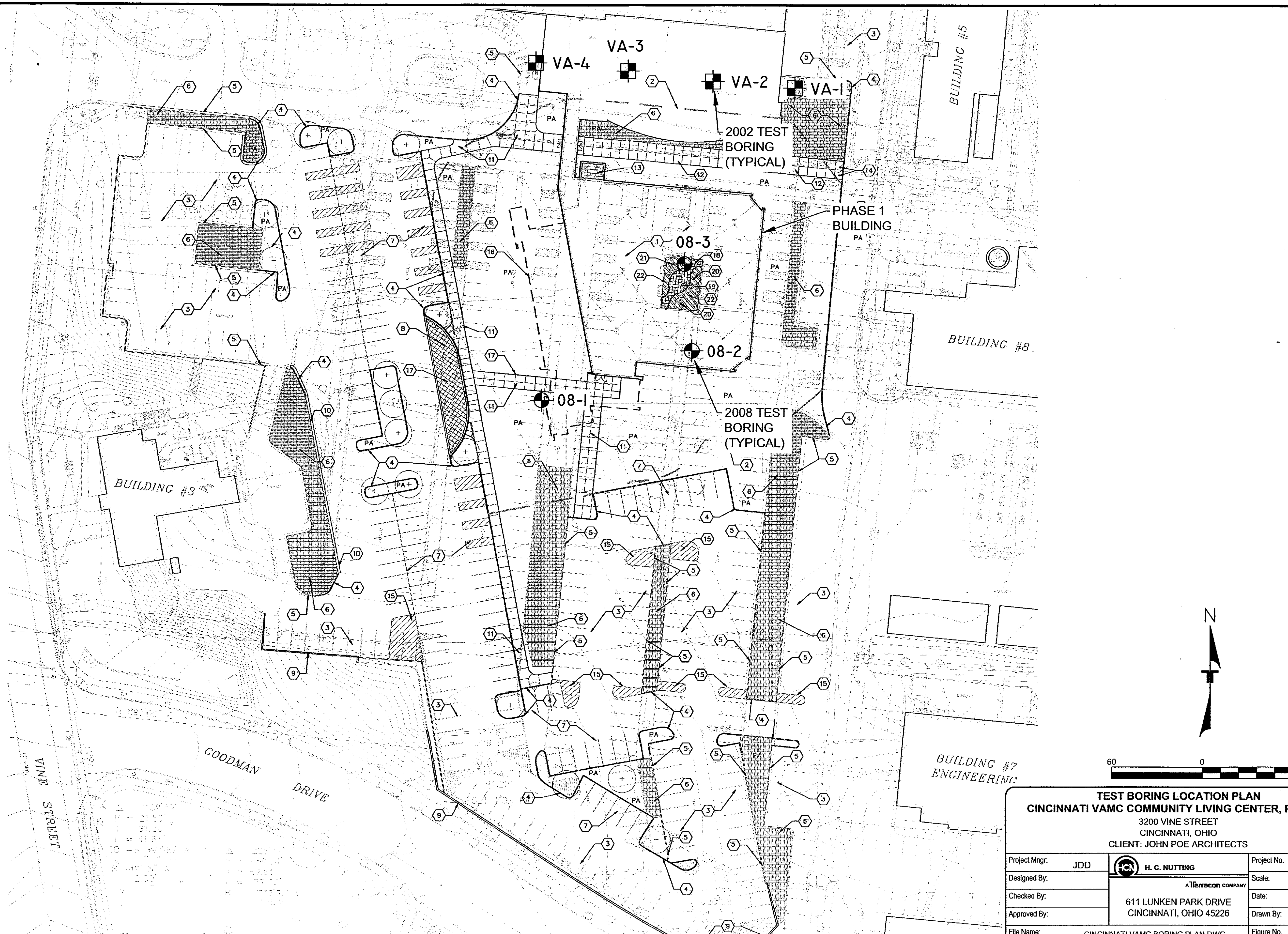
H.C. NUTTING COMPANY
CORPORATE OFFICE – 611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS

DEC. 2008 N1085608

SITE LOCATION MAP

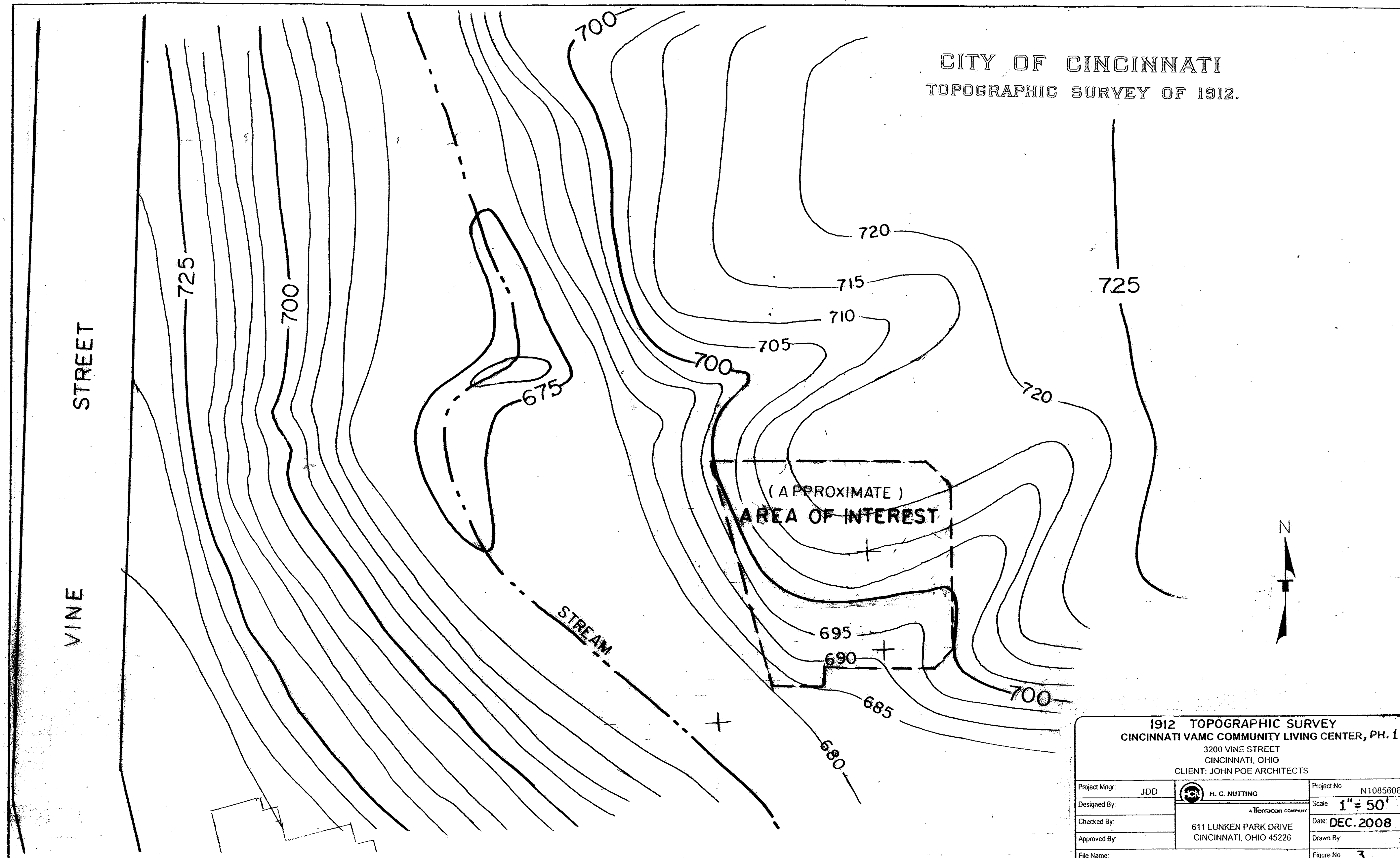
CINCINNATI VAMC COMMUNITY LIVING CENTER, PHASE 1
3200 VINE STREET
CINCINNATI, OHIO
CLIENT: JOHN POE ARCHITECTS

FIGURE 1

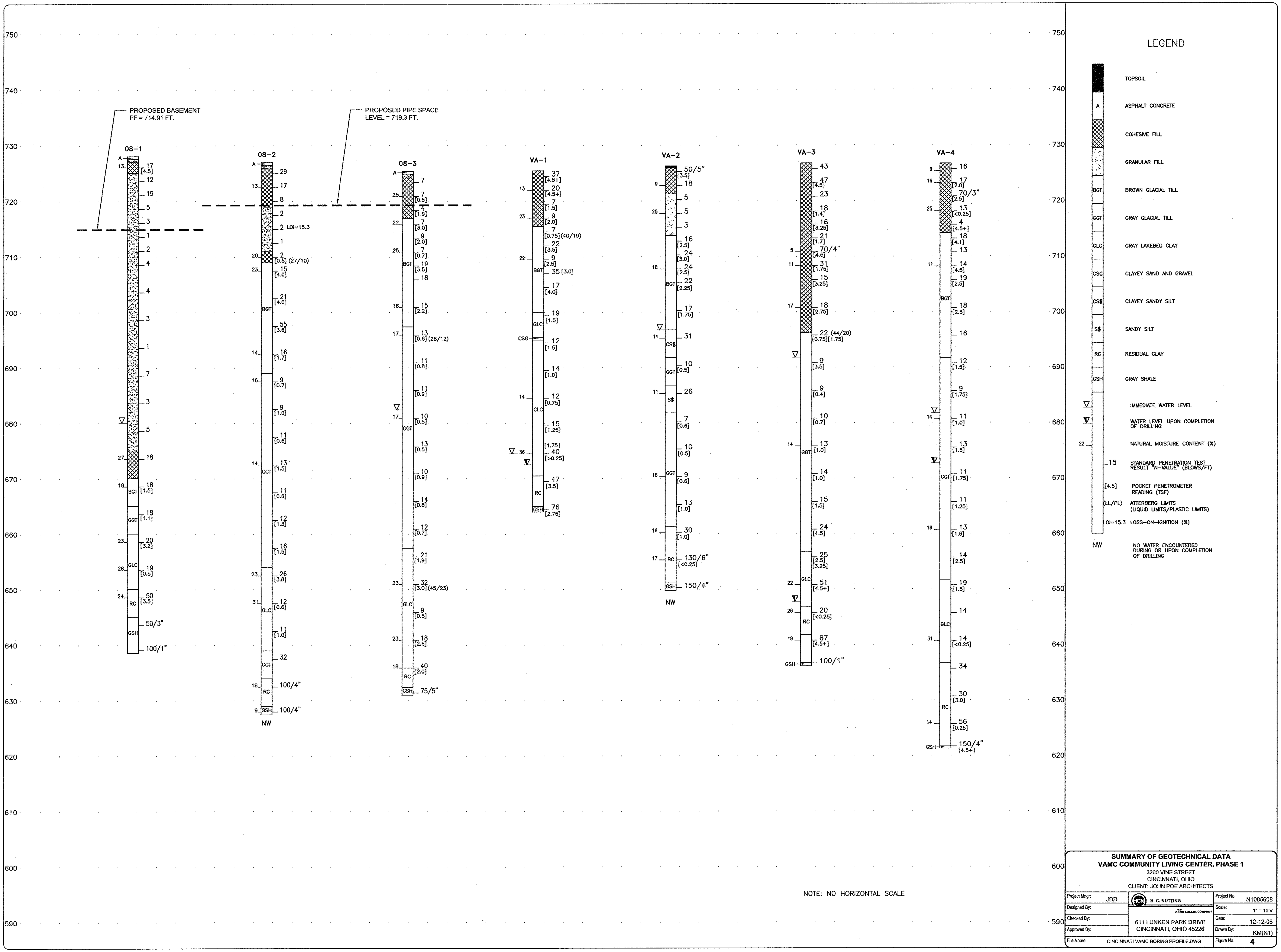


TEST BORING LOCATION PLAN
CINCINNATI VAMC COMMUNITY LIVING CENTER, PHASE 1
3200 VINE STREET
CINCINNATI, OHIO
CLIENT: JOHN POE ARCHITECTS

Project Mngr: JDD	H. C. NUTTING A Terracon COMPANY	Project No.	N1085608
Designed By:		Scale:	1" = 60'
Checked By:	611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	Date:	12-10-08
Approved By:		Drawn By:	KM(N1)
File Name:	CINCINNATI VAMC BORING PLAN.DWG	Figure No.	2

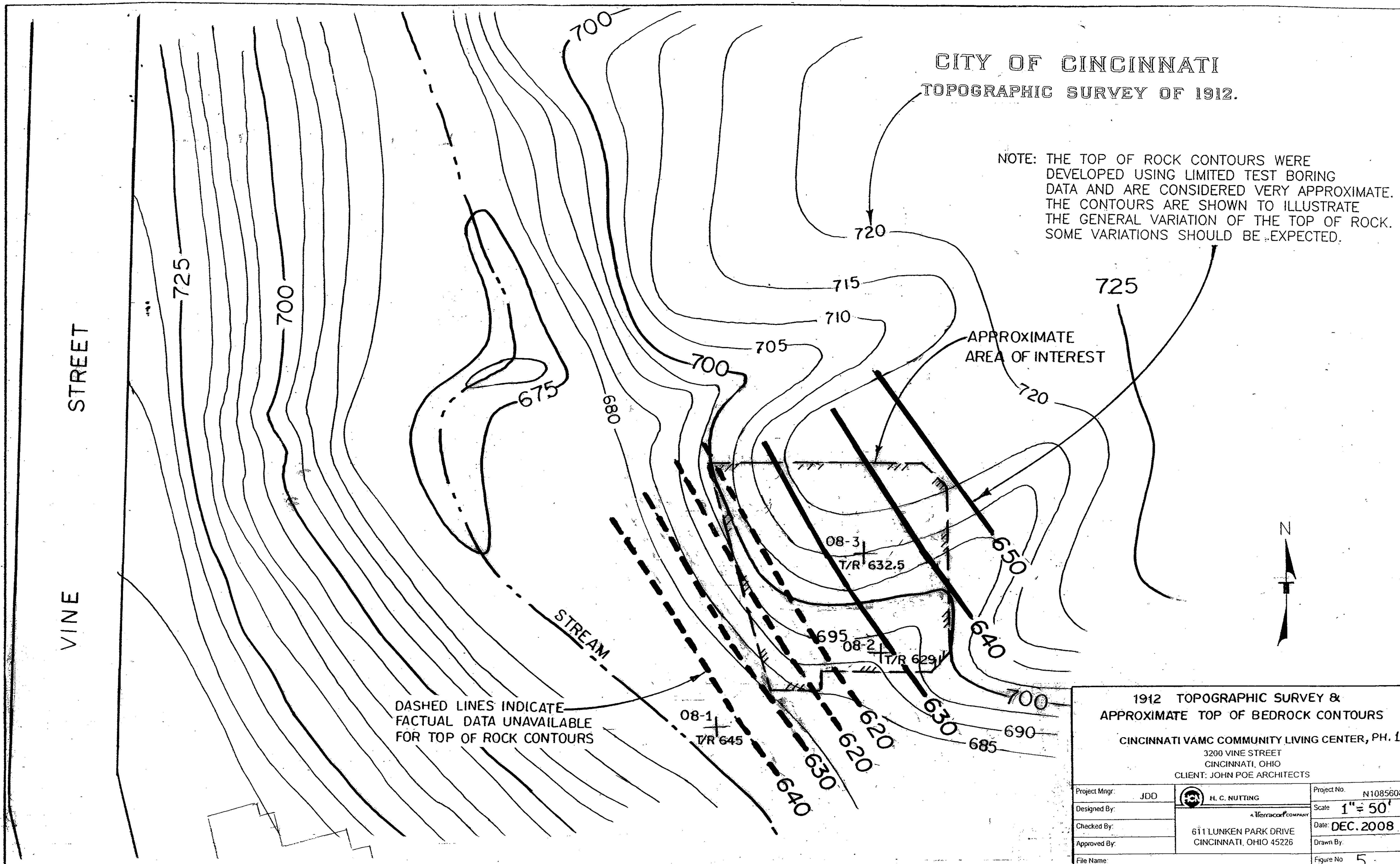


CITY OF CINCINNATI
TOPOGRAPHIC SURVEY OF 1912.
VINE
STREET
725
700
675
700
705
710
715
720
720
725
700
695
690
685
680
STREAM
(APPROXIMATE)
AREA OF INTEREST
N
1912 TOPOGRAPHIC SURVEY
CINCINNATI VAMC COMMUNITY LIVING CENTER, PH. 1
3200 VINE STREET
CINCINNATI, OHIO
CLIENT: JOHN POE ARCHITECTS
Project Mngr: JDD
Designed By:
Checked By:
Approved By:
File Name:
H. C. NUTTING
A Terracon COMPANY
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
Project No. N1085608
Scale 1" = 50'
Date: DEC. 2008
Drawn By:
Figure No. 3



SUMMARY OF GEOTECHNICAL DATA
VAMC COMMUNITY LIVING CENTER, PHASE 1
3200 VINE STREET
CINCINNATI, OHIO
CLIENT: JOHN POE ARCHITECTS

Project Mngr:	JDD	H. C. NUTTING	Project No.	N1085608
Designed By:		A Terracon COMPANY	Scale:	1" = 10'V
Checked By:		611 LUNKEN PARK DRIVE	Date:	12-12-08
Approved By:		CINCINNATI, OHIO 45226	Drawn By:	KM(N1)
File Name:	CINCINNATI VAMC BORING PROFILE.DWG		Figure No.	4


CITY OF CINCINNATI
TOPOGRAPHIC SURVEY OF 1912.
NOTE: THE TOP OF ROCK CONTOURS WERE DEVELOPED USING LIMITED TEST BORING DATA AND ARE CONSIDERED VERY APPROXIMATE. THE CONTOURS ARE SHOWN TO ILLUSTRATE THE GENERAL VARIATION OF THE TOP OF ROCK. SOME VARIATIONS SHOULD BE EXPECTED.
VINE STREET
725
700
675
680
700
705
710
715
720
720
725
APPROXIMATE AREA OF INTEREST
STREAM
DASHED LINES INDICATE FACTUAL DATA UNAVAILABLE FOR TOP OF ROCK CONTOURS
08-1 T/R 645
08-2 T/R 629
08-3 T/R 632.5
695
650
640
630
620
620
620
630
640
700
690
685
N
1912 TOPOGRAPHIC SURVEY & APPROXIMATE TOP OF BEDROCK CONTOURS
CINCINNATI VAMC COMMUNITY LIVING CENTER, PH. 1
3200 VINE STREET
CINCINNATI, OHIO
CLIENT: JOHN POE ARCHITECTS
Project Mngr: JDD
Designed By:
Checked By:
Approved By:
File Name:
H. C. NUTTING
a Terracon COMPANY
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
Project No. N1085608
Scale 1" = 50'
Date: DEC. 2008
Drawn By:
Figure No 5

LOG OF BORING NO. 08-1

CLIENT	ELEVATION REFERENCE
John Poe Architects	**MH Rim, Elevation = 726.88**
SITE	**PROJECT**
Cincinnati, Ohio	**Cincinnati VAMC Community Living Center**

Boring Location: See Attached Test Boring Location Plan

Approx. Surface Elev.: 728.1 ft

Depth, ft	Description	Elev.
0.5	ASPHALT CONCRETE (6")	727.5
1	Granular Base (6")	727
3	Brown lean clay with sand and gravel (FILL), moist - very stiff to hard	725
	Black, dark brown and dark reddish brown cinders with sand, noted glass pieces, coal pieces, metal pieces, coal pieces, and slag (FILL), slightly moist to moist - medium dense to very loose -- medium dense 3 to 8 ft. -- very loose to loose 8 ft. to 38 ft. -- wet below 48 ft.	

SAMPLES					TESTS				
NUMBER	TYPE	DEPTH, ft.	RECOV, in./(RQD %)	BLOWS / 6in. (SPT - N)	WATER CONTENT, %	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNCONFINED STRENGTH, tsf	POCKET PEN, tsf
1	SS	1 - 2.5	10	2-6-11 (17)	13				4.5
2	SS	3.5 - 5	14	8-7-5 (12)					
3	SS	6 - 7.5	18	6-11-8 (19)					
4	SS	8.5 - 10	12	3-3-2 (5)					
5	SS	11 - 12.5	2	1-1-2 (3)					
6	SS	13.5 - 15	6	1-0-1 (1)					
7	SS	16 - 17.5	11	1-1-1 (2)					
8	SS	18.5 - 20	10	1-2-2 (4)					
9	SS	23.5 - 25	13	1-2-2 (4)					
10	SS	28.5 - 30	8	1-1-2 (3)					
11	SS	33.5 - 35	10	1-0-1 (1)					
12	SS	38.5 - 40	12	3-5-2 (7)					
13	SS	43.5 - 45	2	1-1-2 (3)					
14	SS	48.5 - 50	16	2-2-3					

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	∇ 48 Immed.	▼ NW Comp.	
WL	▼ BF After 0 Hrs	▼	
WL	No water used in drilling.		

H. C. NUTTING
A Terracon COMPANY

BORING STARTED	12-8-08
BORING COMPLETED	12-8-08
RIG Central Star	FOREMAN
LOGGED JDD	JOB # N1085608

LOG OF BORING NO. 08-1

CLIENT	ELEVATION REFERENCE
John Poe Architects	**MH Rim, Elevation = 726.88**
SITE	PROJECT
Cincinnati, Ohio	**Cincinnati VAMC Community Living Center**

DESCRIPTION			SAMPLES: NUMBER	TYPE	DEPTH, ft.	RECOV, in./(RQD %)	BLOWS / 6in. (SPT - N)	TESTS: WATER CONTENT, %	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNCONFINED STRENGTH, tsf	POCKET PEN, tsf
							(5)					
Brown lean clay with sand and gravel, noted rock pieces (POSSIBLE FILL), wet - medium stiff	53	675	15	SS	53.5 - 55	14	5-8-10 (18)	27				
Brown SANDY LEAN CLAY, trace gravel (GLACIAL TILL), very moist - stiff	58	670	16	SS	58.5 - 60	18	7-8-10 (18)	19				1.5
Gray LEAN CLAY with sand, noted gravel (GLACIAL TILL), very moist - stiff	63	665	17	SS	63.5 - 65	18	8-8-10 (18)					1.1
Gray CLAY, interbedded silt partings (LAKEBED), moist - very stiff	68	660	18	SS	68.5 - 70	18	3-6-14 (20)	23				3.2
Gray SILTY CLAY, interbedded silt and sand seams (LAKEBED), very moist - soft	73	655	19	SS	73.5 - 75	18	7-7-12 (19)	28				0.5
Blue-gray laminated CLAY, note shale pieces, very moist to moist - very stiff	78	650	20	SS	78.5 - 80	15	22-24-26 (50)	24				3.5
Gray soft SHALE, noted interbedded limestone layers	83	645	21	SS	83.5 - 85	10	47-50/3					
			22	SS	88.5 - 89.5		100/1					
BORING TERMINATED AT 89.5 ft	89.5	638.5										

Depth scale, ft.: 55, 60, 65, 70, 75, 80, 85

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft				
WL	∇ 48	Immed.	▼ NW	Comp.
WL	▼ BF	After 0 Hrs	∇	
WL	No water used in drilling.			

H. C. NUTTING
A Terracon COMPANY

BORING STARTED	12-8-08		
BORING COMPLETED	12-8-08		
RIG	Central Star	FOREMAN	
LOGGED	JDD	JOB #	N1085608

LOG OF BORING NO. 08-2

CLIENT: John Poe Architects	ELEVATION REFERENCE: MH Rim, Elevation = 726.88
SITE: Cincinnati, Ohio	PROJECT: Cincinnati VAMC Community Living Center

Boring Location: See Attached Test Boring Location Plan

Approx. Surface Elev.: 727 ft

Depth (ft)	DESCRIPTION	Elev.
0.5	ASPHALT CONCRETE (6")	726.5
1	GRANULAR BASE (6")	726
8	Brown to grayish-brown lean clay with sand and gravel, noted small brick, coal, and glass pieces (FILL), slightly moist to moist-very stiff -noted rock fragments from 1-3 ft. and 6-8 ft.	719
16	Reddish brown to black fine to coarse cinders, noted small brick, glass, metal, and coal pieces (FILL), moist-very loose	711
18	Brown silty clay, trace sand (POSSIBLE FILL), very moist-soft	709
23	Brown and light brown LEAN CLAY, noted sand and concretions, moist-very stiff	704
38	Brown LEAN CLAY with sand, trace gravel (GLACIAL TILL), moist-very stiff -brown and gray and stiff below 34 ft.	689
	Gray LEAN CLAY, trace sand and gravel (GLACIAL TILL), very moist-medium stiff to stiff	

SAMPLES NUMBER	TYPE	DEPTH, ft.	RECOV, in./(RQD %)	BLOWS / 6in. (SPT - N)	WATER CONTENT, %	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNCONFINED STRENGTH, tsf	POCKET PEN, tsf
1	SS	1 - 2.5	12	2-7-22 (29)					
2	SS	3.5 - 5	11	10-8-9 (17)	13				
3	SS	6 - 7.5	5	7-5-3 (8)					
4	SS	8.5 - 10	5	1-1-1 (2)					
5	SS	11 - 12.5	6	1-1-1 (2)					
6	SS	13.5 - 15	5	WOH-0-1 (1)					
7	SS	16 - 17.5	13	1-1-1 (2)	20	27	10		0.5
8	SS	18.5 - 20	16	4-6-9 (15)	23				4.0
9	SS	23.5 - 25	16	10-10-11 (21)					4.0
10	SS	28.5 - 30	15	9-31-24 (55)					3.6
11	SS	33.5 - 35	18	5-7-9 (16)	14				1.7
12	SS	38.5 - 40	18	4-3-6 (9)	16				0.7
13	SS	43.5 - 45	18	4-4-5 (9)					1.0
14	SS	48.5 - 50	18	3-6-5					0.6

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft				
WL	NW	Immed.	NW	Comp.
WL	BF	at 0 hrs	30	Caved
WL	No water used in drilling.			

H. C. NUTTING
A Terracon COMPANY

BORING STARTED	12-10-08
BORING COMPLETED	12-8-08
RIG Central Star	FOREMAN
LOGGED JDD	JOB # N1085608

LOG OF BORING NO. 08-2

CLIENT: John Poe Architects	ELEVATION REFERENCE: MH Rim, Elevation = 726.88
SITE: Cincinnati, Ohio	PROJECT: Cincinnati VAMC Community Living Center

Depth / Elev.	DESCRIPTION	DEPTH, ft.	NUMBER	TYPE	DEPTH, ft.	RECOV, in./(RQD %)	BLOWS / 6in. (SPT - N)	WATER CONTENT, %	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNCONFINED STRENGTH, tsf	POCKET PEN, tsf
	Gray LEAN CLAY, trace sand and gravel (GLACIAL TILL), very moist-medium stiff to stiff						(11)					
		55	15	SS	53.5 - 55	18	3-6-7 (13)	14				1.5
		60	16	SS	58.5 - 60	18	4-4-7 (11)					0.6
		65	17	SS	63.5 - 65	18	4-6-6 (12)					1.3
		70	18	SS	68.5 - 70	18	4-7-9 (16)					1.5
73 / 654	Gray varved FAT CLAY, trace interbedded silt partings (LAKEBED), moist-very stiff	75	19	SS	73.5 - 75	18	8-10-16 (26)	23				3.8
78 / 649	Gray CLAY with interbedded silt seams (LAKEBED), very moist-medium stiff	80	20	SS	78.5 - 80	18	4-4-8 (12)	31				0.6
		85	21	SS	83.5 - 85	18	2-6-5 (11)					1.0
88 / 639	Gray LEAN CLAY, trace sand, gravel, and limestone pieces and possible cobbles (GLACIAL TILL), moist-very stiff	90	22	SS	88.5 - 90	4	10-10-22 (32)					
93 / 634	Blue-gray CLAY with limestone pieces and cobbles, very moist-medium stiff	95	23	SS	93.5 - 93.8		100/0.3'	18				
98 / 629			24	SS	98.5 - 99.5		100/0.3'	9				
99.5 / 627.5												

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft				
WL	∇ NW	Immed.	▼ NW	Comp.
WL	▼ BF	at 0 hrs	▼ 30	Caved
WL	No water used in drilling.			

H. C. NUTTING
A Terracon COMPANY

BORING STARTED	12-10-08		
BORING COMPLETED	12-8-08		
RIG	Central Star	FOREMAN	
LOGGED	JDD	JOB #	N1085608

LOG OF BORING NO. 08-2

CLIENT	ELEVATION REFERENCE
John Poe Architects	MH Rim, Elevation = 726.88
SITE	PROJECT
Cincinnati, Ohio	Cincinnati VAMC Community Living Center

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: DEPTH, ft.	SAMPLES: RECOV, in./(RQD %)	SAMPLES: BLOWS / 6in. (SPT - N)	TESTS: WATER CONTENT, %	TESTS: LIQUID LIMIT, %	TESTS: PLASTICITY INDEX, %	TESTS: UNCONFINED STRENGTH, tsf	TESTS: POCKET PEN, tsf
	Gray soft SHALE, trace interbedded limestone layers Auger Refusal at 99.5 ft.											
	BORING TERMINATED AT 99.5 ft											

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft				
WL	∇ NW	Immed.	▼ NW	Comp.
WL	▼ BF	at 0 hrs	▼ 30	Caved
WL	No water used in drilling.			

H. C. NUTTING
A Terracon COMPANY

BORING STARTED	12-10-08		
BORING COMPLETED	12-8-08		
RIG	Central Star	FOREMAN	
LOGGED	JDD	JOB #	N1085608

LOG OF BORING NO. 08-3

CLIENT	ELEVATION REFERENCE
John Poe Architects	MH Rim, Elevation = 726.88
SITE	PROJECT
Cincinnati, Ohio	Cincinnati VAMC Community Living Center

Boring Location: See Attached Test Boring Location Plan

Approx. Surface Elev.: 725.4 ft

Depth (strata)	DESCRIPTION	Elev.	DEPTH, ft.	NUMBER	TYPE	DEPTH, ft.	RECOV, in./(RQD %)	BLOWS / 6in. (SPT - N)	WATER CONTENT, %	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNCONFINED STRENGTH, tsf	POCKET PEN, tsf
0.5	ASPHALTIC CONCRETE (6")	725											
1	Granular Base (6")	724.5		1	SS	1 - 2.5	5	1-3-4 (7)					
5.5	Brown to black lean clay with sand, trace cinders and fine gravel, noted pieces of concrete (FILL), moist - soft	720	5	2	SS	3.5 - 5	6	3-3-4 (7)	25				0.5
8.5	Brown with gray and dark brown lean clay, noted sand (POSSIBLE FILL), moist - stiff	717		3	SS	6 - 7.5	13	3-2-2 (4)					1.9
13.5	Brown LEAN CLAY, trace sand, moist - very stiff - noted silt partings below 11 ft.	712	10	4	SS	8.5 - 10	15	3-4-3 (7)	22				3.0
				5	SS	11 - 12.5	13	3-4-5 (9)					2.0
16	Brown SILTY CLAY, noted sand, very moist - medium stiff	709.5	15	6	SS	13.5 - 15	12	1-2-5 (7)	25				0.7
28	Brown LEAN CLAY with sand, trace gravel (GLACIAL TILL), moist - very stiff - noted small limestone pieces and silt seams below 18.5 ft.	697.5		7	SS	16 - 17.5	18	5-8-11 (19)					3.5
			20	8	SS	18.5 - 20	14	7-8-10 (18)					
			25	9	SS	23.5 - 25	18	4-6-9 (15)	16				2.2
	Gray LEAN CLAY, trace sand and gravel (GLACIAL TILL), very moist - medium stiff - noted small limestone pieces and sand partings below 38.5 ft. - soft and wet from 43 ft. to 53 ft.		30	10	SS	28.5 - 30	10	7-6-7 (13)	17	28	12		0.6
			35	11	SS	33.5 - 35	18	4-5-6 (11)					0.8
			40	12	SS	38.5 - 40	18	3-5-6 (11)					0.9
	▽		45	13	SS	43.5 - 45	18	2-4-6 (10)	17				0.5
			50	14	SS	48.5 - 50	13	4-6-7					0.5

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	▽ 43.0 Immed.	▼ NW Comp.
WL	▽ BF After 0 Hrs	▼
WL	No water used in drilling.	

H. C. NUTTING
A Terracon COMPANY

BORING STARTED	12-8-08		
BORING COMPLETED	12-8-08		
RIG	Central Star	FOREMAN	
LOGGED	JDD	JOB #	N1085608

LOG OF BORING NO. 08-3

CLIENT	ELEVATION REFERENCE
John Poe Architects	MH Rim, Elevation = 726.88
SITE	**PROJECT**
Cincinnati, Ohio	Cincinnati VAMC Community Living Center

Depth, ft.	Description	Elevation
	Gray LEAN CLAY, trace sand and gravel (GLACIAL TILL), very moist - medium stiff - noted small limestone pieces and sand partings below 38.5 ft. - soft and wet from 43 ft. to 53 ft.	
68		657.5
	Gray varved LEAN CLAY, trace interbedded silt partings (LAKEBED) moist - stiff - very stiff below 73.5 ft.	
78		647.5
	Gray varved LEAN CLAY (LAKEBED), very moist - soft	
83		642.5
	Blue-gray FAT CLAY, noted fine gravel, moist - very stiff - very moist below 88 ft.	
89.5		636
	Gray laminated CLAY, moist - very stiff	
93		632.5
	Gray soft SHALE	
94.5		631
	BORING TERMINATED AT 94.5 ft	

SAMPLES: NUMBER	TYPE	DEPTH, ft.	RECOV, in./(RQD %)	BLOWS / 6in. (SPT - N)	TESTS: WATER CONTENT, %	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNCONFINED STRENGTH, tsf	POCKET PEN, tsf
				(13)					
15	SS	53.5 - 55	18	3-5-5 (10)					0.9
16	SS	58.5 - 60	18	4-6-8 (14)					0.8
17	SS	63.5 - 65	18	5-5-7 (12)					0.7
18	SS	68.5 - 70	18	7-9-12 (21)					1.9
19	SS	73.5 - 75	18	11-14-18 (32)	23	45	23		3.0
20	SS	78.5 - 80	18	2-4-5 (9)					0.5
21	SS	83.5 - 85	18	5-7-11 (18)	23				2.6
22	SS	88.5 - 89.5	12	3-20					2.0
22A	SS	89.5 - 90	6	20	18				
23	SS	93.5 - 94.5	11	39-75/5					

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 43.0 Immed.	▼ NW	Comp.
WL	▼ BF After 0 Hrs	▼	
WL	No water used in drilling.		

H. C. NUTTING
A Terracon COMPANY

BORING STARTED	12-8-08		
BORING COMPLETED	12-8-08		
RIG	Central Star	FOREMAN	
LOGGED	JDD	JOB #	N1085608

H.C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

John Poe Architects
Cincinnati VAMC Community Living
Cincinnati, Ohio
W.O. #N1085608

TABLE I: CLASSIFICATION TEST DATA

Boring No.	Sample No. (SS)	Depth (Ft.)	Moisture Content (%)	Loss On Ignition (%)	Atterberg Limits		
					Liquid Limit (%)	Plastic Limit (%)	Plasticity Index
08-1	1	1-2.5	12.7				
	15	53.5-55	27.3				
	16	58.5-60	18.7				
	18	68.5-70	22.8				
	19	73.5-75	28.0				
	20	78.5-80	23.9				
08-2	2	3.5-5	12.6				
	5	11-12.5		15.3			
	7	16-17.5	19.5		27	17	10
	8	18.5-20	23.4				
	11	33.5-35	13.7				
	12	38.5-40	15.7				
	15	53.5-55	14.4				
	19	73.5-75	23.3				
	20	78.5-80	31.1				
	23	93.5-95	18.2				
	24	98.5-99.5	9.1				
08-3	2	3.5-5	24.8				
	4	8.5-10	21.5				
	6	13.5-15	24.5				
	9	23.5-25	16.1				
	10	28.5-30	17.3		28	16	12
	13	43.5-45	16.7				
	19	73.5-75	22.7		45	22	23
	21	83.5-85	23.4				
	22A	89.5-90	17.8				

H. C. NUTTING

A Terracon COMPANY

HCN Division Office
611 Lunken Park Drive
Cincinnati, OH 45226
Phone 513.321.5816
Fax 513.321.0294
www.hcnutting.com

SAMPLE DISPOSITION

Unless other arrangements are made with H. C. Nutting Company (HCN), all soil and rock core samples collected during the course of this work will be disposed of 30 days after our report or lab test result submittal.

If the client wishes to avoid sample disposal in 30 days, other arrangements can be made, including any of the following:

1. The samples may be picked up by the client's representative from HCN's office, as prescheduled with HCN. The pick up date must precede the 30-day limit described above.
2. The samples can be shipped to the client by HCN. All costs associated with shipping shall be borne by the client.
3. The samples can be stored by HCN at a cost borne by the client. This cost will be based on the type of samples stored (boxes of soil sample jars, rock core boxes, etc.) and the duration of storage. Specific needs for sample storage beyond 30 days shall be detailed in the contract at agreed upon rates.

Requested Alternate Action:

______Samples to be picked up by Client
(arrangements will be coordinated with Laboratory Manager)

______Samples to be shipped to: ______________________________
(costs borne by client) ______________________________
____
____

______Samples to be stored by HCN at negotiated rates

Acknowledgment:
Company:______________________________
Name:______________________________
Signature:______________________________
Date:______________________________

Please return this form to: H. C. Nutting Co. 611 Lunken Park Dr. Cincinnati, OH 45226
Attn: Laboratory Manager
Phone: (513) 321-5816, Fax: (513) 321-0294